Soils and Foundations 2014;54(4):523–543

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Lessons for countermeasures using earth structures against tsunami obtained in the 2011 Off the Pacific Coast of Tohoku Earthquake

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Received 24 October 2011; received in revised form 26 June 2013; accepted 3 September 2013
Available online 23 August 2014

Abstract

Since many infrastructures, such as sea walls, sand beaches, forests, etc., were severely damaged or destroyed by the tsunami that occurred due to the 2011 Off the Pacific Coast of Tohoku Earthquake, it is said that the resistant functions of the above structures against tsunami attacks did not perform well. However, some structures are known to have resisted the tsunami, based on field surveys conducted after the earthquake by the authors and others; and thus, the resistant functions of those infrastructures against tsunami should be estimated more properly. This paper focuses on earth structures, including river levees and road embankments, both damaged and undamaged, at 13 sites in Miyagi Prefecture, Chiba Prefecture, and Ibaraki Prefecture. They have been investigated through field surveys and other related data, such as satellite photographs taken before and/or after the tsunami. Furthermore, 10 dug pools, eroded by the flood stream on the back side of sea walls and banks during the tsunami, are also investigated to clarify their effects against tsunami attacks for use as future hardware countermeasures. Based on the above field investigations, several important lessons on hardware countermeasures against tsunami, using earth structures, are discussed. And, performance-based design concepts for reconstruction after this earthquake and for the reduction of future tsunami damage are discussed and proposed.

Keywords: The 2011 Off the Pacific Coast of Tohoku Earthquake; Tsunami; Earth structure; Dug pool; Hardware countermeasures; Performance-based design (IGC: C8/E7/H4/H7)

1. Introduction

Many infrastructures, such as sea walls, sand beaches, and forests were severely damaged by the tsunami flood which occurred due to the 2011 Off the Pacific Coast of Tohoku Earthquake. However, post-earthquake field surveys, conducted by the authors and others (Tokida and Koizumi, 2011a–c; Tokida and Tanimoto, 2011) from April 30 to May 3, July 8 to 10, and September 10 to 12, 2011, revealed that some structures were able to resist the tsunami attack. It is now a major concern, therefore, whether or not these damaged structures can satisfactorily resist future tsunami attacks.

The main objective of the current study is to investigate the effectiveness of such structures in resisting and/or reducing damage from tsunami. This study is based on field surveys which were conducted on earth structures, including river levees and road embankments, both damaged and undamaged, at 13 sites in Miyagi Prefecture, Chiba Prefecture, and Ibaraki Prefecture. Other relevant data, including pre- and post-earthquake satellite images, are used. Moreover, 10 dug pools, eroded by the flood stream of the tsunami on the back side of sea walls and banks, are also investigated.

Based on the discussions presented herein, important lessons on the use of earth structures as hardware countermeasures against tsunami attacks are learned.

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Peer review under responsibility of The Japanese Geotechnical Society.

http://dx.doi.org/10.1016/j.sandf.2014.07.001

A new performance-based design approach is also proposed for use in designing post-tsunami reconstruction works and countermeasures against future tsunami attacks.

2. Earth structure and damage

2.1. Objective earth structure

Typical structures used to reduce tsumani damage may be classified into 3 areas, breakwater, detached breakwater, manmade reefs, natural reefs in sea areas, sand beaches, sea walls, wave dissipation blocks, sand dunes, fishing ports in coastal areas, forests, lagoons, lakes, canals, river levees, road embankments, and railway embankments in inland areas.

Based on the results of field surveys conducted by the authors and others (Tokida and Koizumi, 2011; Tokida and Tanimoto, 2011, 2012), it was revealed that these structures can contribute to a reduction in tsunami height, flood depth, run-up height, and flow velocity and/or water pressure.

In this paper, focus is placed on earth structures and banks, which are not covered by structures such as blocks and which do not touch the water area daily, to investigate their resistance characteristics against tsunami for use as future countermeasures. River levees and road embankments are also included, as they are similar to the above conditions.

The selected objective earth structures of the current study are located on the Sendai Plain in Miyagi Prefecture and along the Asahi Coast in Chiba Prefecture, as shown in Figs. 1 and 2, respectively. The main characteristics of these structures are summarized in Table 1. The structures in Miyagi Prefecture total 9 and include 2 river levees, 1 road embankment, 2 banks, and 4 artificial banks, whereas those in Chiba Prefecture include 3 banks at 3 sites. Table 1 also includes a summary of a sand dune, located in Ibaraki Prefecture, which has eroded slopes. It is thought to be a good a reference when considering the resistance features of earth structures.

The earth structures considered in this study are characterized, as shown in Table 1, in terms of the structure height, the physical condition of the slope, the slope, and other factors. This characterization is based on field surveys and other available information. As the conditions of a tsunami, such as tsunami height and run-up height, can generally not be measured during the tsunami, these factors for the Sendai Plain are estimated by the tsunami traces and on the basis of the height of the earth structures and the ground level measured in the field, based on the tsunami height of 10 m, which is generally considered on the Sendai Plain.

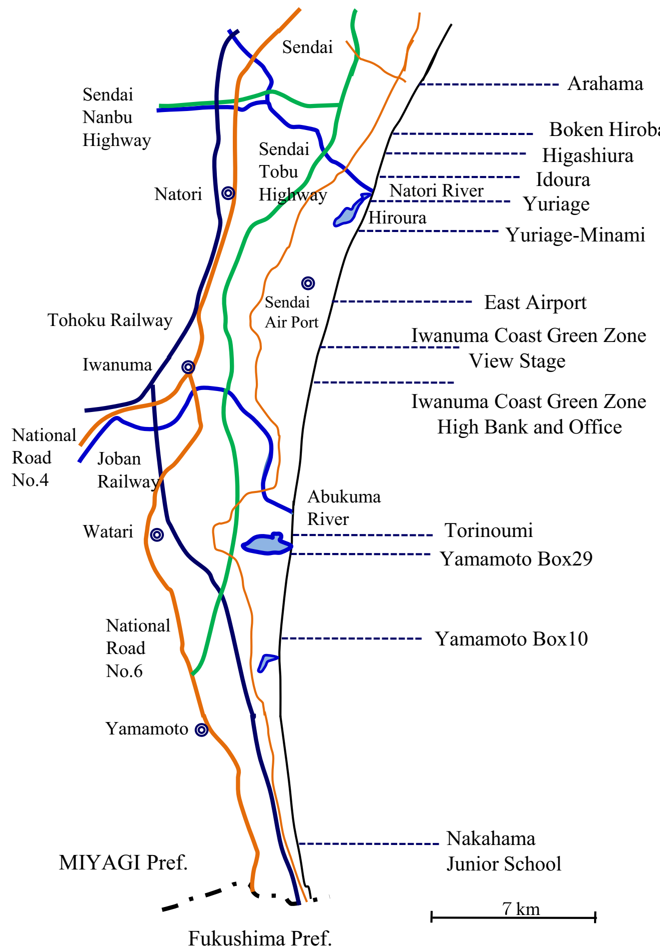

Fig. 1. Field survey sites on Sendai Plain in Miyagi Prefecture.

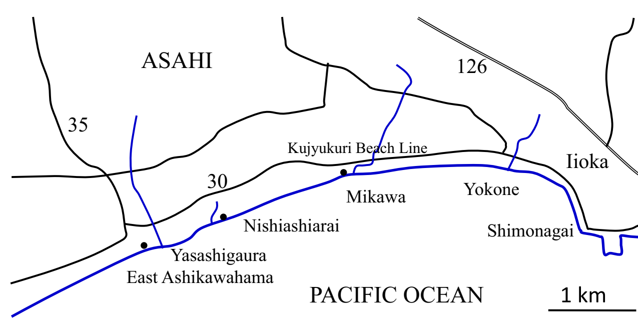

Fig. 2. Field survey sites on Asahi Coast in Chiba Prefecture.

2.2. Damage to earth structures

The damage induced by the tsunami and/or the conditions before the earthquake are explained for each earth structure in Table 1 with the lessons learned.

2.2.1. Example 1: Idoura

Photo 1 is a satellite image of a levee newly constructed along the existing Teizan Canal at Idoura in the Wakabayashi area of Sendai City. The new levee is approximately 3.9 m high and 1.5 km long and is located around a lagoon whose width is approximately 200 m. The purpose of this levee is to reinforce the Teizan Canal whose height is approximately 2 m. The photo shows that the coastal zone was washed away by the tsunami.

Although the estimated overflow depth of the tsunami, shown in Table 1, is 3.85 m, which is a little less than that of the new levee, the lawn on the surface of the front slope of the levee was not eroded, but the sand that was washed away

Table 1
Objective earth structures and conditions.

No.	Type of bank	Site of bank		Content of earth structure				Condition of Tsunami	
				Height (m)	Condition of slope	Slope of front slope	Others	Flooded depth (m)	Overflowed depth (m)
1	River Levee	Miyagi Pref.	Idoura	Front 3.9 back 3.2 Teizan Canal 5.0	Lawn	1: 2.7	Front slope L10.7 m/H3.9 m, crest width 6.9 m, back slope L10.9 m/H3.2 m, as-pavement	7.75*	3.85*
2			Higashiura	Back 2.7 Teizann Canal 4.5	Lawn	1: 3	Levee: L9.7 m/H2.1 m, crest width 6.5 m, not paved	6.8*	4.1*
3	Road embankment		Takenohana	6**	Block grass low tree	–	4 lanes, block: H2 m	1.6	None
4	Bank		Yuriage	5.6	Bare:soil	1: 2.7	Front slope L15 m/H5.6 m, crest width 24 m, L150 m, crest surface: netted	6.1**	0.5**
5			Yuriage Minami–East Airport	1.5	Grass, low tree	1 : 7	Front slope W11 m, back slope W4 m, surface layer 0.6 m, covered net	6.1**	4.6**
6	Artificial Bank		Bohken Hiroba	14.9	Tree, grass	Upper 1 :4 lower 1: 5	Ship shape W50 m, L400 m	10.55–13.8	None
7			Mt. Hiyoriyama	6.55	Grass	1: 2.5	Top: $12 \times 20m^2$	8.65	2.1
8			Iwanuma Coast Green Zone: view stage	9.8	Low tree	Front 1:2 back 1:4	Top: φ10 m, circle	6.8	None
9			Iwanuma Coast Green Zone: High Bank	9.5	Grass	1: 2.8	Top: 20 m × 30 m base: 60 m × 280 m	3.9	None
10	Bank	Chiba Pref.	East Ashikawahama	1.5–2.5	Grass	1: 1**	Block wall: H3.5 m, non parapet, slope shoulder W3.5 m + walking way W3 m,	2.5**	1.0–0**
11			Nishiashiarai	2.0	Grass	1: 1**	Vertical sea wall H2 m, palapet H0.7 m, cycling way W3 m	3.2以上**	1.2以上**
12			Mikawa	2.0	Grass	1: 1**	Vertical sea wall H2 m, palapet H0.9 m, crest W2.5 m, back slope H1.5–2 m, cycling way W3 m	2.5–3.0**	0.5–1.0**
13	Sand Dune	Ibaragi Pref.	Oarai Coast	Parking 15 m**, toilet 13 m**	Grass, toe: bare	1: 3.3–1: 10**	General Section 1: 3.3, Reef Section 1 : 10	Run up height: parking 9**, toilet 13.8**	Non at parking, toilet 0.8

*Estimate based on the tsunami height of 10 m.
**Estimated.

by the tsunami settled a little. The arrow in Photo 2 shows the direction of the leading wave; this arrow indicates the direction of the leading wave throughout the paper. Photo 3 shows the crest of the levee, which is paved with asphalt 0.1 m in thickness. Although the asphalt was partially removed, as shown in the photo, the levee remained functioning.

Photo 4 shows the erosion at the back slope and the toe due to the tsunami; it indicates that the erosion at the back side of the levee is more severe than that at the front slope and the crest. Photo 5 shows a water pool eroded by the flood stream (such a pool is henceforth referred to as a ‘dug pool’) and a slope failure. The failure level, in terms of the level of performance reduction from the engineering viewpoint, appears to be low.

Photo 1. New river levee at Idoura on April 6.

Photo 2. Front slope after tsunami on May 2 (Idoura).

Photo 3. Crest after tsunami on May 2 (Idoura).

The levee, indicated by a circle in Photo 1, is approximately 100 m long. It eroded and approximately a depth of 1 m of the crest was washed away by the concentration of tsunami flow. Nonetheless, the whole stability of the levee was maintained.

The dug pool shown in Photo 5 is clearer in the satellite image in Photo 6. It is seen that the forest located behind the dug pool was not completely washed away, but remained after the tsunami. On the other hand, it is interesting to note that the forest at the back side of the levee, where there is no dug pool, no longer exists; it was washed away by the tsunami floodwater. This observation on the difference in damage level of the forest represents an important lesson and implies that dug pools are effective in reducing the force of a tsunami.

2.2.2. Example 2: Higashiura

The levee at Higashiura, shown in Photo 7, is located approximately 600 m from Idoura. It was also constructed to reinforce the Teizan Canal. In Photo 7, dug pools can be seen at the back side of the levee; it should be noted that there are

Photo 4. Back slope after tsunami on May 2 (Idoura).

Photo 5. Dug pool and slope failure on May 2 (Idoura).

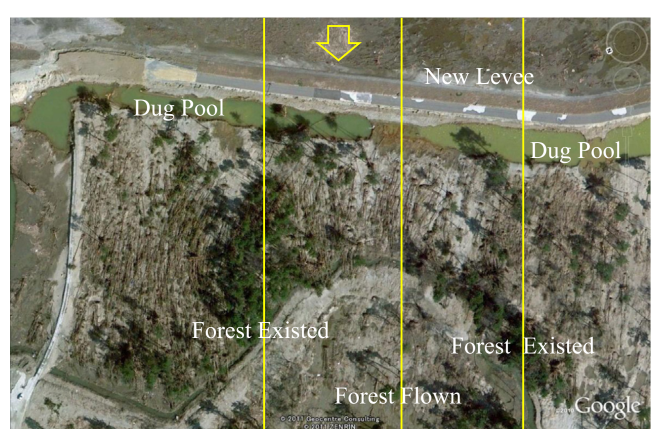

Photo 6. Dug pool and forest on April 6 (Idoura).

levees with and without dug pools. A relationship similar to that for Idoura's site, between the existence of a dug pool and the forest damage, can be seen in Photo 7.

As shown in Photo 8, the new reinforcing levee, approximately 2.1 m high, exists on the bank of the Teizan Canal. Although the lawn planted on the front slope was washed away by the tsunami, the erosion is limited to the surface. Moreover, as shown in Photo 9, the back slope is also eroded only on the surface and the dug pool can be clearly seen. Nonetheless, the performance of the levee is maintained.

In Photo 10, in the direction of the leading wave, it can be seen that the forest in front of the dug pool almost remained, whereas the forest was swept away where there was no dug pool.

The remaining forest, shown on the left side of Photo 7, is approximately 225 m long and 150 m wide. Several houses at the back side of this forest appear to be flooded to a depth of 3.5 m, but were not washed away, as shown in Photo 11.

These observations indicate that dug pools are effective in protecting forests. Subsequently, forests contribute to the protection of existing houses by reducing the energy of the tsunami. This represents an important approach for effective countermeasures against future tsunami attacks.

2.2.3. Example 3: Takenohana and Imaizumi

The tsunami struck the inland of the Sendai Plane far away from the coastal line. The Sendai Tobu Highway is located from about 2.3 km in Watari Town to about 4.2 km in Wakabayashi, far from the coast line between Sendai City and Yamamoto Town.

The Sendai Tobu Highway is mainly an embankment. Photos 12 and 13 show the conditions around the embankment after the tsunami on April 30 at Takenohana and Imaizumi, respectively, which are located on the left side of Natori River, about 2.5 km from the river mouth on the mountain side and on the sea side, respectively.

The embankment structure, including culvert boxes and an elevated bridge, as shown in Photo 12, is approximately 6 m

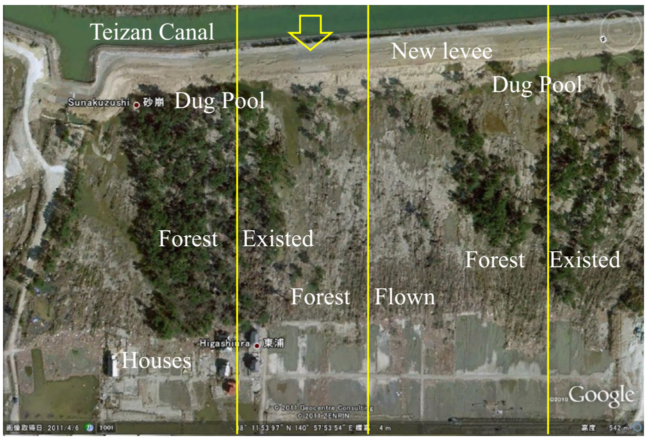

Photo 7. Dug pool, forest and house on April 6 (Higashiura).

Photo 8. Eroded new levee on April 30 (Higashiura).

Photo 9. Back slope after tsunami on April 30 (Higashiura).

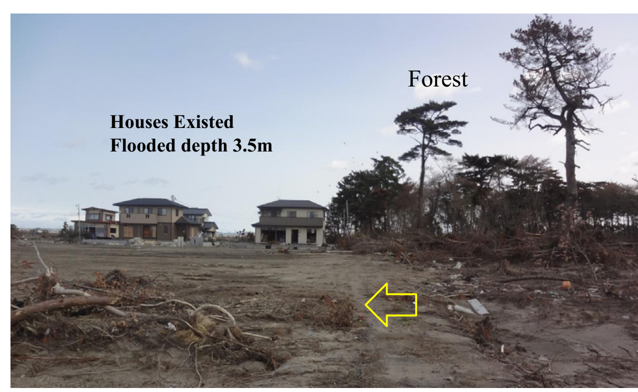

Photo 11. Residual houses at back of forest on April 30 (Higashiura).

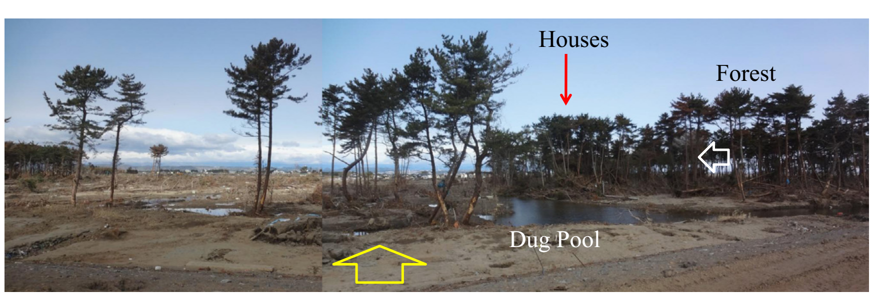

Photo 10. Comparison of eroded background of levee with uneroded background on April 30 (Higashiura).

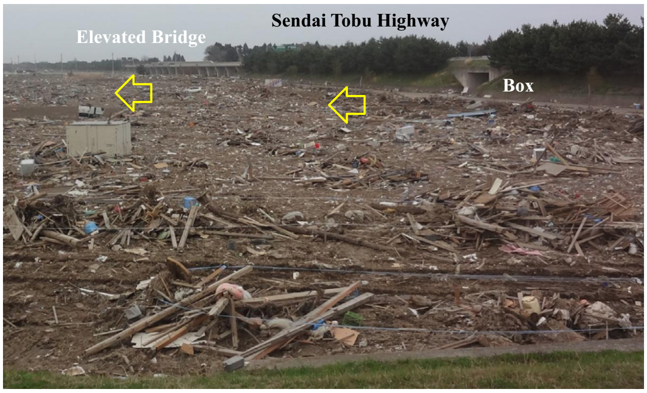

Photo 12. View on mountain side of embankment on April 30 (Takenohana).

Photo 13. View on sea side of embankment on April 30 (Imaizumi).

high with the slope toe reinforced with concrete blocks. The embankment was flooded, and the flooded depths measured at the sea side and the mountain side are 1.9 m and 1.15 m, respectively. The flooded height at the mountain side appears to be almost two thirds that at the sea side. In addition, the debris swept away on the sea side around the embankment, shown in Photo 13, appears to be similar to or a little worse than that on the mountain side, as shown in Photo 12.

As mentioned above, the embankment appears to be effective against the tsunami in this earthquake. However, the following three conditions should be confirmed for the case of future tsunami, because boxes, elevated bridges, and rivers (see Photo 14) are sensitive structures and most likely cannot effectively stop or reduce tsunami attacks.

Condition 1: overflow at road embankment

Although the tsunami did not flow over the embankment of the Sendai Tobu Highway in this earthquake, because it is far from the coast, much concern should be given to the tsunami overflow for embankments close to the coast.

Condition 2: overflow from river

Although the tsunami did not overflow from the Natori River, which is close to the embankment of the Sendai Tobu Highway, in this earthquake, overflow from the mountain side of the river may be experienced in a future tsunami.

Condition 3: horizontal alignment of embankment

The embankment in the longitudinal direction is almost parallel to the coastal line; this rendered the embankment effective against the tsunami. Thus, it would be useful to pay attention to the horizontal alignment of embankments when considering potential attacks by future tsunami. The closer the horizontal alignment of an embankment is to the direction perpendicular to the flow of the tsunami, the more effective the embankment will be in stopping the flow of the tsunami.

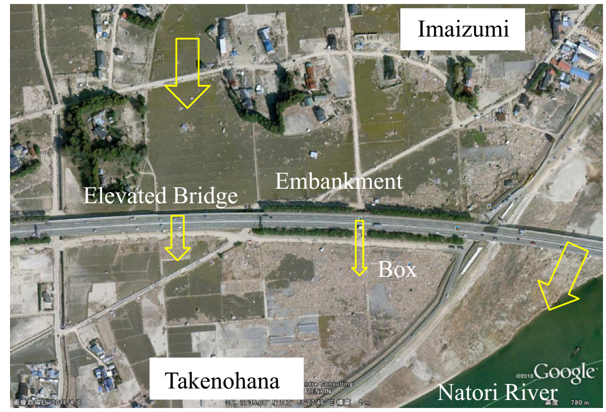

Photo 14. Overview of highway embankment on April 6 (Takenohana and Imaizumi).

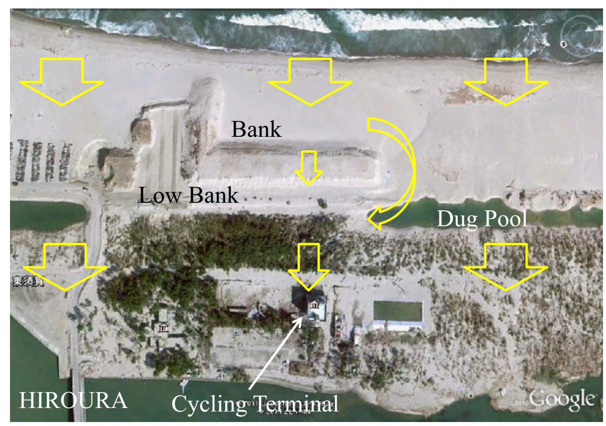

Photo 15. Temporary bank on beach after tsunami on April 6 (Yuriage).

2.2.4. Example 4: Yuriage

At the sand beach, located on the east side of the Yuriage Fishing Port, a sand bank was found during the field survey on July 9. As shown in Photo 15, this bank is approximately 5.6 m high and 150 m long; the crest is approximately 24 m wide and the front slope on the sea side is approximately 1:2.7. Photo 16 (Japan Coast Guard, 2011) shows the bank just after the tsunami attack. It can be seen that the tsunami wave around the bank almost stopped and that the forest at the back side of the bank is flooded slightly. Although the objective of the bank, when it was newly constructed, was not to function as a countermeasure against tsunami, it was able to effectively resist the tsunami of March 11, as shown in Photo 16. The photo also shows that the tsunami goes around both edges of the bank and into the forest, and that the flooded depth at the cycling terminal in the forest is approximately 4.3–5 m.

It appears that this depth was reduced because of the existence of the bank. It is believed that the bank would have been more effective against the tsunami if its length had been more than 150 m along the sand beach.

The existence of a green-colored net covering the bank, seen in Photo 16, is verified in a satellite image before the earthquake on August 14, 2009. However, it appears that the net was not covered with surface soils when the tsunami struck. The field survey of July 9 revealed that the net was mostly swept away, but there were some pieces remaining. The crest of the bank was slightly flooded, and the estimated flooded depth is approximately 0.5 m (see Table 1). This estimate is based on two observations of the crest: (1) the remaining pieces of the net, and (2) the existence of a small 0.3-m-high pine tree.

On the other hand, Photos 17 and 18 show the conditions of the front and the back slopes of the bank, respectively, and indicate that both slopes were slightly eroded by the tsunami. Photo 18, in addition, shows on back side the existence of a low bank, which is 1.6 m high and 12.8 m in base width. The surface of this low bank is not eroded, as shown in the photo. This is attributed to the flood reduction by the high front bank.

Photo 19, which shows a view at the right edge of the bank shown in Photo 15, reveals that trees had fallen in the direction from the left side to the right side of the photo. This means that the tsunami went around the edge of the bank. The remaining net can also be seen in Photo 19.

Therefore, a lesson that can be learned, on the basis of the aforementioned, is that although the bank was able to reduce the overflowed depth of the tsunami, there was an overflow of approximately 0.5 m.

2.2.5. *Example 5: East Yuriage–East Airport*

On the basis of the field survey, the low bank shown in Photo 18 is located at the boundary between the sand beach and the forest from the beach at Yuriage in Example 4 to the east-side beach of Sendai Airport. The dug pool with or without water, shown in Photo 15, can be seen at the back of the low bank.

Photo 20 shows a low bank located approximately 900 m from the bank of Example 4. A section of this bank is shaped like a triangle with a height of 1.5 m and front and back slopes of approximately 1: 7.3 and 1: 2.7, respectively. The slope surface is covered with grass and pine trees, as shown in Photo 21. The surface layer is reinforced with nets installed to

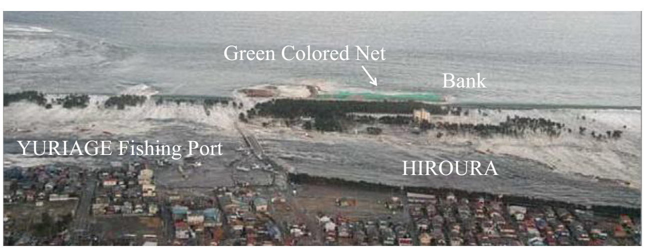

Photo 16. Bank attacked by tsunami on March 11 (Yuriage).

Photo 17. Front slope of bank on July 9 (Yuriage).

Photo 18. Back slope of bank and background on July 9 (Yuriage).

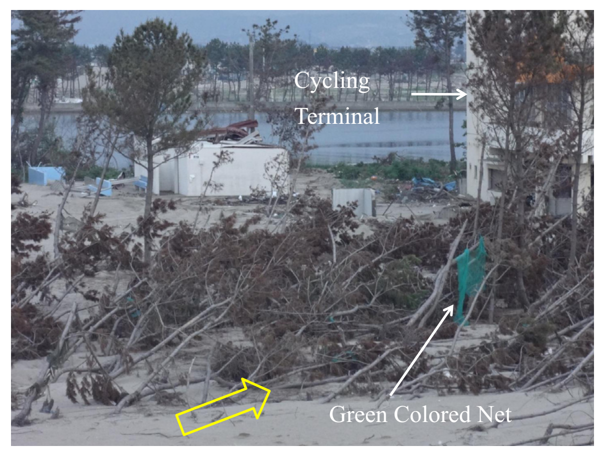

Photo 19. Trace of direction of tsunami flow on July 9 (Yuriage).

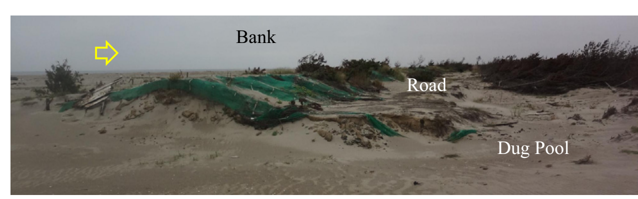

Photo 20. Eroded low bank and dug pool on September 11 (East Yuriage–East Airport).

Photo 21. Grass not eroded at front slope on September 11 (East Yuriage–East Airport).

approximately 0.3 m in depth. These nets appear effective in protecting the surface against tsunami-induced erosion.

Assuming that the flooded depth is 6.1 m, like that for the bank in Example 4, the overflow depth at the crest of the current low bank is estimated to be approximately 4.6 m. Since the depth of overflow was slightly larger, the simply paved road, approximately 3 m wide, and the forest were eroded by the tsunami. Furthermore, the dug pool, which is approximately 1.5 m deep and 10–15 m in base width, was partially formed and the back and/or front slopes of the low bank were eroded, and then the whole body was partially washed away, as shown in Photo 22.

As for the reinforcement methods of the banks against attacks by tsunami, Photo 21 indicates the effectiveness of the reinforcement using synthetic fiber nets.

2.2.6. Example 6: Boken Plaza

An artificially made high bank was found in the Wakabayashi area of Sendai City. Field surveys were conducted at this site on May 2 and September 10. This high bank was constructed as a public ground for an adventure play named Boken Plaza in this paper and located approximately 350 m from the coastal line, as shown in Photos 23 and 24. The bank is shaped like a ship and is approximately 400 m long and 80 m wide. The top of the bank is located on the sea side and the bank is sloped to the west, i.e., the mountain side. With the aid of a plate set at the view stage, constructed on top of the bank, it was found that the height of the bank is 15.89 m. As the height at the top, measured from the surrounding ground surface, is 14.9 m, the estimated flooded depth is 10.55 m; this is based on the flooded trace observed by the authors on the surface of the bank. Shibayama et al. (2011), however, reported that the flooded trace was 1.1 m below the top; and thus, the flooded depth was 14.79 m. For a height at the top of 14.9 m, a flooded depth of 13.8 m can be estimated.

It is concluded, therefore, that the flooded depth at this high bank fell in the range of 10.55–13.8 m.

From the geotechnical engineering viewpoint, when the stability of a high bank is concerned, the erosion and/or failure of the bank should be considered. The eroded conditions on both sides and the front of the bank, where the tsunami severely collides or flows, may be discussed as follows.

Photos 25 and 26 show the eroded conditions on May 2 on the side and at the front of the bank after the tsunami, respectively. The slopes of both sides are 1: 4–1: 5 and/or similar to the slope of the river levee; each side is eroded slightly. Photo 27 clearly shows the erosion of a walkway at the front of the bank. It is seen that the damage level is low and can be repaired easily; there is no a geotechnical problem.

Photo 22. Eroded and washed away section of low bank on September 11 (East Yuriage–East Airport).

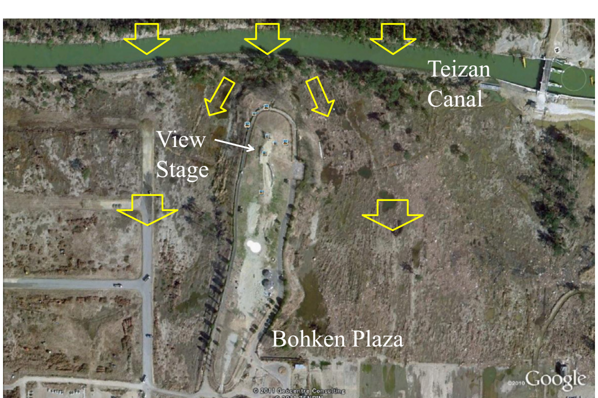

Photo 23. Boken Plaza and surroundings on April 6.

Photo 24. High bank at Boken Plaza on May 2.

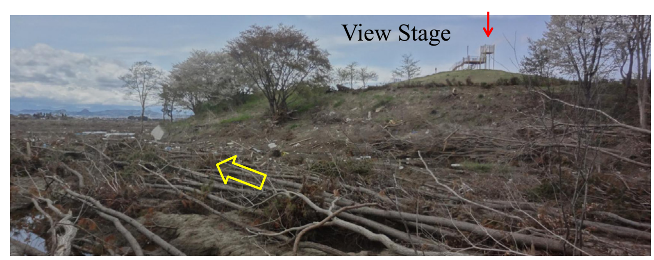

Photo 25. Side of bank blown around by tsunami on May 2 (Boken Plaza).

Photo 26. Front slope facing sea side of bank on May 2 (Boken Plaza).

2.2.7. Example 7: Mt. Hiyoriyama Hill

The Yuriage area of Natori City was severely damaged by the tsunami, as shown in Photo 16. Mt. Hiyoriyama, located in the Yuriage area, can be referred to from the geotechnical engineering viewpoint. Photo 28 shows Mt. Hiyoriyama, which appears as a small manmade bank. It is 6.55 m high and its top forms a flat 12×20 m^2 stage.

The bank was blown over by the tsunami and the flooded depth at the top was reported to be 2.1 m (Shibayama et al., 2011). Therefore, the flooded depth at this bank can be estimated to be 8.65 m.

Photos 28 and 29 show the whole view from the mountain side and a close view from the sea side, respectively. It appears that the surface of this bank was covered by grass and some trees when the tsunami struck. The measured slope is approximately 1: 2.5, and the erosion of the slope and top of this bank appears to be very small.

2.2.8. Example 8: view stage at Iwanuma Coastal Green Zone

A small manmade bank that acts as a view stage was found in the Iwanuma Coastal Green Zone close to the coast. Photo 30 shows a view of this bank. It is 9.8 m high, the top is flat and circular in shape with a diameter of approximately 10 m, the front slope is 1: 2, and the back slope is 1: 4. Low trees are planted on the surface of the bank. On the basis of the tsunami trace at the surface, the flooded depth was measured and found to be approximately 6.8 m. The tsunami did not flow over this bank and the erosion at the surface appears to be very small.

The observations from Examples 6 to 8 reveal that the corresponding banks were not so severely eroded by the tsunami that their performance would be reduced.

Photo 27. Steps on front slope in Photo 26 on May 2 (Boken Plaza).

Photo 28. Hiyoriyama Hill from mountain side on July 9.

2.2.9. Example 9: high bank at Iwanuma Coastal Green Zone

A high manmade bank, shown in Photo 31, was found in the field survey on May 1 at the Iwanuma Coastal Green Zone; a detailed survey was conducted on September 12. The mountain side of the bank to the west faces the Teizan Canal, and the sea side to the east faces a forest which is located at the back side of the sea wall. The bank consists of two layers, as shown in Photo 32. The lower layer is 2.6 m high above the base ground at the forest, and the upper layer is 6.9 m high. Thus, the bank is 9.5 m high above the base ground. The base of the upper layer is approximately 60×280 m^2, whereas the top, which is a flat stage, is approximately 20×30 m^2.

The top stage is paved with blocks and has been settled and cracked, as shown in Photo 33, because of the seismic ground

Photo 29. Hiyoriyama Hill from sea side on July 9.

Photo 30. View stage at Iwanuma Coastal Green Zone on July 8.

Photo 31. High bank close to Teizan Canal on September 12 (Iwanuma Coastal Green Zone).

Photo 32. Slope on sea side on September 12 (Iwanuma Coastal Green Zone).

Photo 33. Cracks at top of high bank on September 12 (Iwanuma Coastal Green Zone).

Photo 34. Cracks on south slope on September 12 (Iwanuma Coastal Green Zone).

Photo 35. Trace of sand boil on September 12 (Iwanuma Coastal Green Zone).

motion before the tsunami attack. The cracks can be clearly observed in the slope shown in Photo 34, and soil liquefaction can also be confirmed because traces of sand boiling can be seen at the lower side of the slope, as shown in Photo 35.

The flooded depth at the office house shown in Photo 32, approximately 30 m from the bank, was measured to be 1.3 m deep; and thus, the flooded depth from the base ground was 3.9 m.

Therefore, it may be concluded that the flooded depth at the office house was reduced by the lower layer of the bank.

Furthermore, because the height of the upper layer of the bank is 6.9 m, it is assessed that this layer was not over washed over by the tsunami, but the tsunami flow went to the back ground around the edge of the bank. Photo 31 shows that the guard rails at the back side of this bank, which are set on the river levee of Teizan Canal, still partially exist. This most likely indicates that the bank was able to reduce the force of the tsunami. As this bank was not flooded over by the tsunami and the soil liquefaction occurred slightly, the damage and cracks at the top of the bank and the soil liquefaction did not reduce the resistance of the bank against the tsunami flow. However, because a bank cannot resist a tsunami flow when it settles or collapses, and as a consequence, loses its height and/or shape, it is necessary to confirm that it is seismically stable, and thus, can resist future tsunami attacks.

This discussion implies that banks can reduce tsunami attacks; however, much attention should be given to their seismic stability.

2.2.10. Example 10: East Ashikawahama

The site is located along the Asahi Coast in Chiba Prefecture approximately 350 km from the epicenter of the main shock. The sloped sea wall is made of blocks approximately 3.5 m high, the crest is 3 m wide, and the neighboring cycling road is 3 m wide. At the back side of the cycling road, there is a manmade bank. This bank is 2 m high and its slope is a little steeper than 1: 1. For a tsunami height of 6 m, such as that measured at the neighboring site of Yasashigaura, an overflow depth at the crest of the sea wall of approximately 2 m is estimated, and therefore, an overflow depth at the crest of the bank of approximately 0.5 m.

Photo 36 shows the condition at the crest of the bank on April 23 and reveals that the crest was not eroded and the surface of the planted bank could be protected against the shallow overflow of about 0.5 m deep. Photo 37 shows trees planted at the back side of the bank, these trees are tilted a little

Photo 36. Crest of bank by low overflow on April 23 (East Ashikawahama).

Photo 37. Forest by low overflow on April 23 (East Ashikawahama).

Photo 38. Bank with different height on April 23 (East Ashikawahama).

but were not washed away. This implies that banks will not be severely eroded if the overflow of a tsunami is small, and plants can be used to function as surface protection.

Photo 38 shows the bank at the back side of the sea wall which is 1.5–2 m high. It is important to note that the height of this bank is irregular; this is attributed to the change in overflow depth according to the change in bank height. In this case, the estimated overflow depth is 0–0.5 m, and the lower crest represents the weak point in resisting the tsunami. Photo 39 shows a certain part of Photo 38 where the walkway was constructed to pass through the bank. It also shows the edge of the walkway eroded where the tsunami was concentrated.

On the basis of this discussion, it is reasonable to conclude that the tsunami was concentrated at the weak points of the bank, such as the shallow crest and the open space. Therefore, these weak points should be considered in the design of hardware countermeasures including the software ones.

2.2.11. Example 11: Nishiashiarai

Photo 40 shows the left side of the Nonaka River and a bank existing along the sea wall. The bank was cut and an open space, a river mouth, was created. It is seen that the edge of the bank's slope was eroded by the concentration of the tsunami. A cycling bridge, called Hinode Bridge, located approximately 50 m from this site, was washed approximately 8.5 m away.

The sea walls on both sides of this river mouth are vertical concrete walls, 2 m high, with a parapet, 0.7 m high. At the back side of the sea wall, there is a 3-m-wide cycling road and

Photo 39. Pass crossing bank in Photo 38 (East Ashikawahama).

Photo 40. Eroded edge at river mouth on April 23 (Nishiashiarai).

Photo 41. Trees eroded at toe of back slope on April 23 (Nishiashiarai).

a 2-m-high manmade bank, as shown in Photo 40. For a tsunami height greater than 4.5 m and a sea level equal to the lower end level of the sea wall, overflow depths at the top of the parapet and the cycling road can be estimated as greater than 2.5 and 3.2 m, respectively. Thus the overflow depth at the crest of the bank can be estimated as greater than 1.2 m, as shown in Table 1.

Photo 41 shows the eroded condition at the back slope of the bank. It can also be seen that the back slope, the back toe, and the back ground are slightly eroded. It is may be concluded, therefore, that a 2-m-high bank can resist a 1-m-deep overflow from the 1.2-m minimum overflow depth mentioned above.

2.2.12. Example 12: Mikawa

This site is similar to the site in Example 11. As shown in Photo 42, a vertical sea wall that is 2 m high with a 0.9-m-high parapet neighbors a 3-m-wide cycling road and a 2-m-high manmade bank.

Photo 43 shows the front slope of the bank whose crest is 2.5 m wide and back slope is 2–1.5 m high. It is estimated that the lower edge of the sea wall is approximately 0.9–0.4 m high above sea level. Since the tsunami swept over the bank shown in Photo 43, the tsunami height can be estimated as greater than 4.5 m. Thus, the estimated overflow depth is approximately 0.5–1 m, as shown in Table 1. This estimate takes into account the eroded condition shown in Photo 44 and other conditions.

Photo 44 shows the back slope of the bank and reveals that the surface of the slope is not eroded. This implies that the 1.5–2-m back slope is not eroded by an overflow depth of approximately 0.5–1 m.

A further lesson that can be learned, on the basis of Examples 10–12 at Asahi Coast which involve sea walls and banks of steep and non-eroded front slopes, is that the steeper the front slope, the easier and more effective it is to stop and to prevent the rise-up of the tsunami.

Photo 42. Vertical wall of sea wall at Mikawa on April 23.

Photo 43. Front slope erode by low overflow on April 23 (Mikawa).

Photo 44. Back slope eroded by low overflow on April 23 (Mikawa).

2.2.13. Example 13: Oarai Coast

Since natural sand dunes are similar to banks in resisting tsunami, from the engineering viewpoint, a typical sand dune along the Oarai Coast in Ibaraki Prefecture that was attacked by the tsunami in this earthquake was selected and surveyed for the purposes of this study.

Sand dunes can be classified into two types: (1) a front-slope dune, and (2) a front- and back-slope dune. The sand dune at the Oarai Coast is the front-slope type, and thus, the run-up of the tsunami is an issue of interest from the geotechnical engineering viewpoint on the erosion at the front slope of banks. On the other hand, the run-up and overflow of the tsunami at a front- and back-slope dune are issues of interest for the erosion at the front and back slope of banks.

As their slope is almost the same as their natural angle of repose, sand dunes usually appear stable; the lower side of the front slope is usually eroded by tidal waves, and this renders them steep.

The slope of a natural sand dune is different or variable according to the location at the coast. Photos 45 and 46 show a general site along the sand beach and a unique site at the reef coast, respectively. The former is designated as Site A and the latter is designated as Site B in this paper. The average front slopes for Sites A and B are estimated as 1: 3.3 and 1: 10, respectively.

The tsunami height on the Oarai Coast in this earthquake is not known. However, the maximum tsunami height along this coast can be estimated as 4 m, because the tsunami height at Oarai Sea Port and Nakaminato Sea Port, adjacent to the Oarai Coast in the direction of south and north, respectively, was approximately 3.5–4 m.

Considering the flooded traces by the tsunami shown in Photos 47 and 48, the run-up heights of the tsunami at Sites A and B were measured in the field survey and found to be approximately 9 m and 13.8 m, respectively. This indicates that the run-up height on the gentle slope was greater than that

Photo 45. General view at Site A on June 4, slope of 1 3.3 (Oarai Coast).

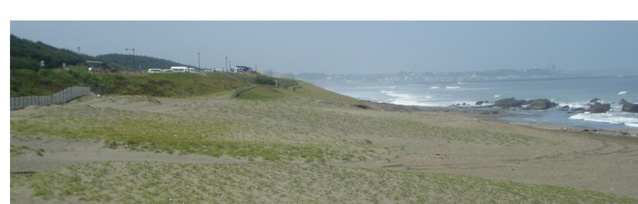

Photo 46. General view at Site B on June 4, gentle slope of 1: 10 (Oarai Coast).

Photo 47. Run-up height at Site A on June 4 (Oarai Coast).

Photo 48. Run-up height at Site B on June 4 (Oarai Coast).

Photo 49. Plant zone eroded by tsunami on June 4 (Oarai Coast).

on the steep one. Photos 49 and 50 show the eroded condition at the lower part of each sand dune. The erosion shown in these photographs was most likely experienced because of the tsunami in this earthquake. However, the maximum height of the eroded zone appears to be approximately 1 m, this could not induce a severe slope failure.

Considering the erosion due to the tsunami and/or the general tidal wave at the sand dune, a lesson that may be learned is that the erosion of the front slopes of sand dunes may occur because of a tsunami, but the level of induced damage for the whole body of a sand dune may be similar to, smaller, or much smaller than that because of a general tidal wave.

3. Dug pool and characteristics

3.1. Objective dug pool

It is very remarkable and interesting that the very large water pools which were dug by the tsunami flood and remained after the earthquake at the back side of the sea walls and banks could be observed at almost every site along the coast of the Sendai Plane. These dug pools are similar to those which appeared during the river flood.

Photo 50. Lower part of slope eroded on June 4 (Oarai Coast).

Photo 51. Dug pool at background of concrete sea wall at Arahama Kita on July 9.

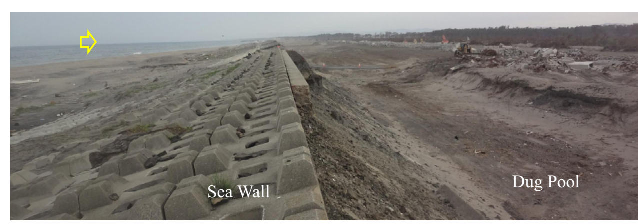

Photo 52. Dug pool at background of block-laid sea wall at east-side of Sendai Airport on July 8.

The typical and unique examples of the Higashiura and Idoura sites, presented in Section 2.2, clearly show the effect of dug pools in reducing the force of the tsunami. Photos 51 and 52 show other examples of dug pools for cases of a concrete sea wall and a block-laid sea wall, respectively.

In this section, the fundamental characteristics of dug pools are investigated. Ten dug pools are selected and their characteristics, such as the scale of the trenches, tsunami conditions, and structural conditions of the sea wall and banks, are investigated.

Furthermore, the sites with and without dug pools at Idoura are investigated in more detail.

The objective sites, selected based on the field surveys, are summarized in Table 2. The size of the dug pool in terms of the width (B), the depth (D), the eroded area (A), the overflowed depth (H_0) at the parapet of the sea walls or the crest of the banks by the tsunami, the structure conditions in

Table 2
Conditions of selected dug pools.

Objective No.	Site	Dug pool Width B (m)	Depth D (m)	Area A (m^2)	Tunami Overflowed depth H_0 (m)	Banke or sea wall Height of back slope H_B (m)	Height of front slope H_F (m)
1	Arahama Kita Wakabayashi	19.9	1.9	22	5.7*	2.6	2.3
2	Arahama Kita Wakabayashi	12.2	2.2	20	6.8*	1.4	1.2
3	Higashiura	13.6	3.4	34	4.1*	2.7	2.1
4	Idoura	16.8	4.6	54	3.85*	3.2	3.9
5	Yuriage Minami	12.5	1.5	19	4.6**	1.5	1.5
6	East Air Port	8.8	2.2	17	4.6**	3.0	1.5
7	Clean Center Iwanuma Coastal Green Zone	11.8	2.8	29	2.8*	2.8	5.2
8	Arahama Kita Watari	13.1	3.4	37	3.05:	2.6	2.0
9	Arahama Minami Watari	26.3	4.1	88	3.05*	2.7	2.0
10	Yoshidahama Minami	16.0	3.7	48	5.5*	3.4	1.5

*Tsumani height is assumed to be 10 m.
**This is estimated based on the overflowed depth at Yuriage Bank.

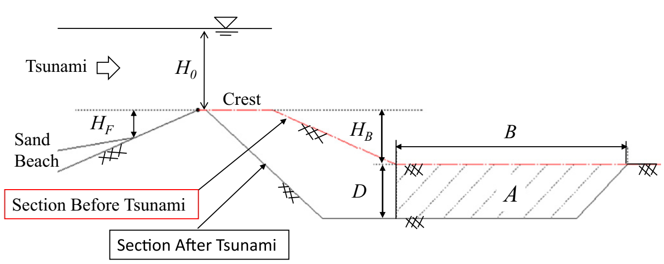

Fig. 3. Characterization of dug pools.

terms of the height (H_B) of the back slope of the sea wall or bank, and the height (H_F) of the front slope are estimated.

These parameters, B, D, A, H_0, H_B, and H_F, are defined as shown in Fig. 3. The overflowed depths of Examples 4 and 5 are estimated on the basis of the overflowed height at Yuriage Bank, shown in Table 2, whereas those of the other 8 sites are estimated on the basis of a tsunami height of 10 m, which is generally assumed on the Sendai Plane.

3.2. General characteristics of dug pool

On the basis of the results in Table 2, the relationships between B and D, H_B and D, and H_B and A are discussed as shown in Figs. 4–6, respectively.

At sites nos. 1 and 9 in Table 2, the sea walls were locally destroyed and apparently sedimented again after the tsunami. However, at the other 8 sites, the sea walls and banks were partially eroded. The relationships are discussed herein for only the sites of partial erosion.

The average relationship between B (m) and D (m) can be represented by Eq. (1.1), and the approximate upper limit can be represented by Eq. (1.2).

$$D = 0.23B \tag{1.1}$$

$$D = 0.27B \tag{1.2}$$

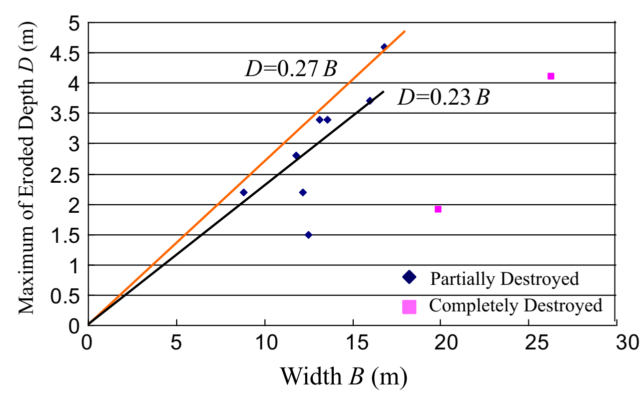

Fig. 4. Relationship between B and D.

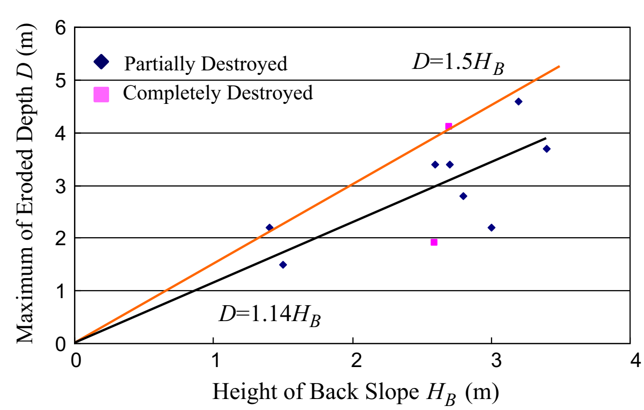

Fig. 5. Relationship between H_B and D.

The average relationship between H_B (m) and D (m) can be represented by Eq. (2.1), and the approximate upper limit can be represented by Eq. (2.2).

$$D = 1.14H_B \tag{2.1}$$

$$D = 1.5H_B \tag{2.2}$$

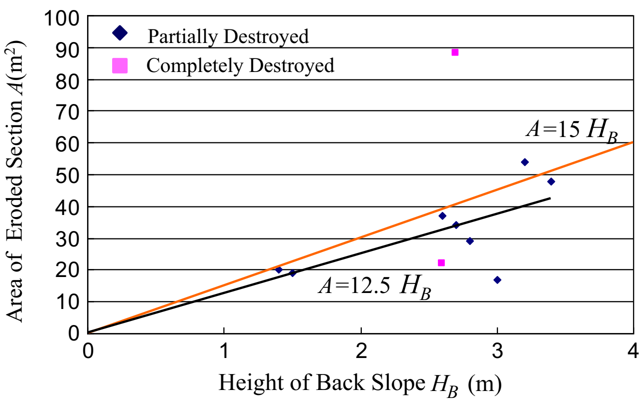

Fig. 6. Relationship between H_B and A.

The average relationship between H_B (m) and A (m^2) can be represented by Eq. (3.1), and the approximate upper limit can be represented by Eq. (3.2).

$$A = 12.5H_B \tag{3.1}$$

$$A = 15H_B \tag{3.2}$$

The relationships imply that the width, depth, or area of dug pools can be simply estimated on the basis of the height of the back slope at the objective sea wall or bank. Moreover, they can be used in the future design of sea walls or banks.

Eqs. (1.1), (1.2), (2.1), (2.2), (3.1) and (3.2) are estimated on the basis of the data measured on the Sendai Plane, where the plane coast is almost straight and generally consists of sand beach, sea wall, and forest. Therefore, they can be used to roughly estimate the conditions of the dug pools on the Sendai Plane for the current tsunami of March 11, 2011.

3.3. Details of dug pool at Idoura

As explained in Section 2.2, the tsunami flow at the back side of the bank was probably influenced by the existence of the dug pool. To make manmade pools function as future countermeasures against tsunami, like dug pools caused by a tsunami flood, a detailed field investigation on the dug pool conditions at Idoura was conducted on September 10–12, 2011.

This investigation included measurements by static cone penetration tests and dynamic cone penetration tests. Soil samples were also taken around the dug pool. The layout of the tests and samples is shown in Fig. 7. A view of the site from the crest of the river levee is shown in Photo 53. The direction of this view is indicated in Fig. 7.

Distributions of the modified N-value of the static cone penetration tests and dynamic ones are shown in Fig. 8 in three categories: 0–4, 4–10, and 10–30. The measurements at Sections A and B, shown in Fig. 7, are combined together in the figure; A designates measurements at Section A, whereas B designates those at Section B. The modified N-value of the dynamic cone penetration tests and the static ones are

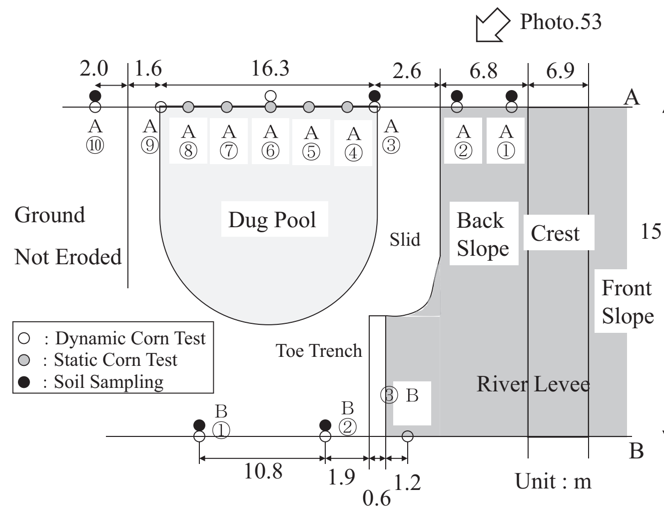

Fig. 7. Layout of measurement at Idoura.

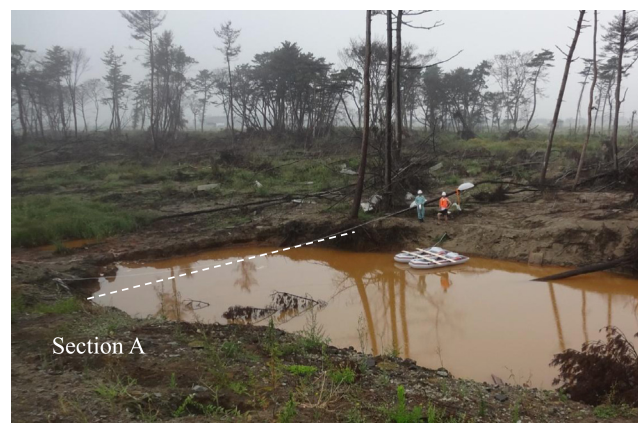

Photo 53. Field survey on dug pool at Idoura on September 11.

calculated using Eq. (4) and (5.1), (5.2), respectively.

$$N = N_d/1.5 \tag{4}$$

where N_d=number of falls per 10 cm of driving, and N=modified N-value.

$$q_c = 5q_u \tag{5.1}$$

$$q_u = 12.5N \tag{5.2}$$

where q_c=cone resistance of the static cone penetration tests (kN/m^2) and q_u=unconfined compressive strength (kN/m^2).

Compared with the results in Fig. 8 at Sections A and B in terms of two categories, 0–4 and 4–10, which represent loose soils, it may be reasonable to say that the corresponding soils above the bottom of the dug pool at Section A sedimented a little after the tsunami. In other words, the original ground eroded because of the tsunami, and this was followed by a little sedimentation after the tsunami. A sedimentation of approximately 0.6 m, for instance, can be found at the site of A⑥S in Fig. 8; this represents 13% of the maximum eroded depth of 4.6 m.

Photo 54 shows the ground surface at the back side of Section B and indicates that the ground surface is not severely

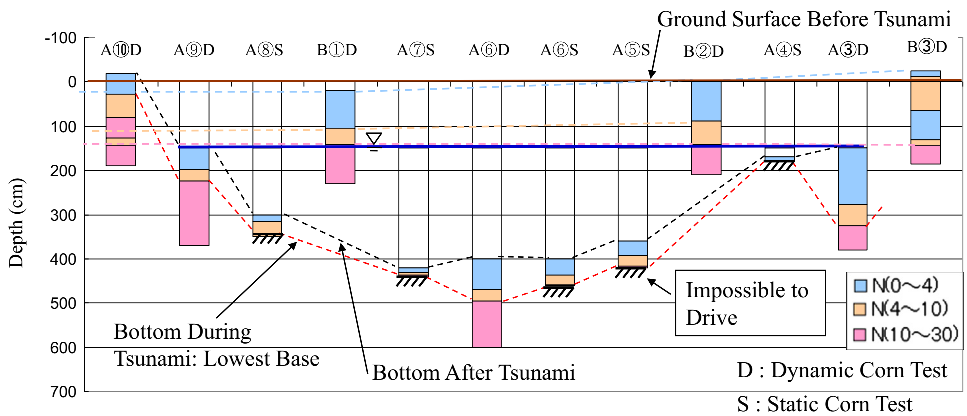

Fig. 8. Soil profiles at Sections A and B, and distribution of depth of dug pool.

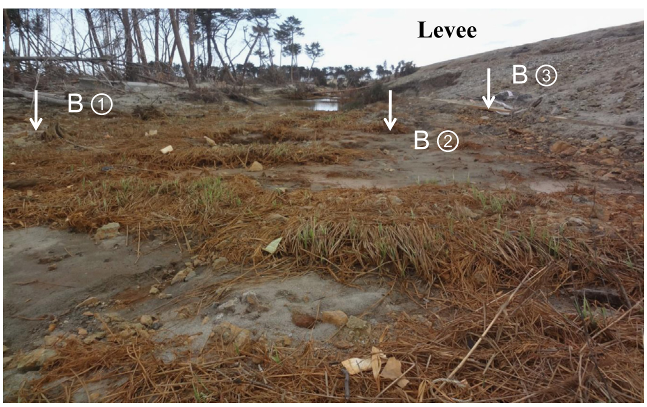

Photo 54. View of field survey at Idoura on May 2.

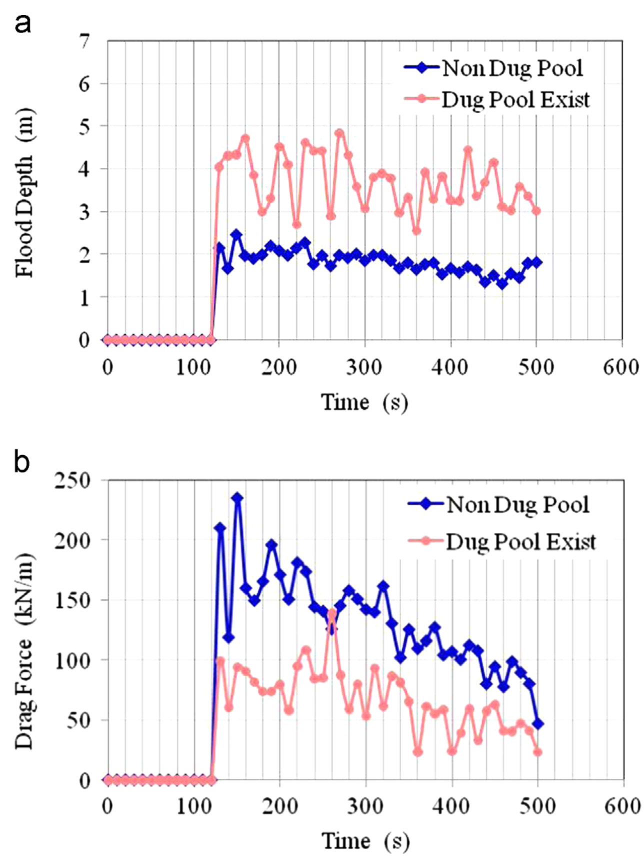

Fig. 9. Example comparing effects of dug pool. (a) Flood depth at back site of dug pool. (b) Drag force at back site of dug pool.

eroded. This is attributed to the existence of gravel soils and plants which cover the ground surface and render it a little harder than that at Section A. Detailed results on the dug pool at Idoura were reported by Tanimoto et al. (2011).

As mentioned above in this section, because the flow velocity and the flood depth at the back side of the dug pools could not be measured during the tsunami and are difficult to estimate, the relationship between the structural conditions of the dug pools in terms of the width and depth is difficult to discuss in detail at present. However, we have tried to apply an analytical tsunami simulation model at the site of Idoura. It has been found that contrary to the increase in flood depth at the back side of the dug pool, the corresponding drag force that is related to the damage level of structure decreases, as shown in Fig. 9 (Tanimoto et al., 2012).

4. Flooded depth and reduction

To quantitatively estimate and/or check the effectiveness of tsunami hard countermeasures, such as earth structures, it is necessary to evaluate the flooded depth because of the tsunami at the back ground. For the purposes of this study, information on the flooded depth and the distance from the coastal line were collected during the field survey and from related references by other researchers. This information is summarized in Table 3 for 18 sites. Information for 14 sites was surveyed by the authors, for 1 site was obtained from Takahashi et al. (2011), and for 3 sites was obtained from Shibayama et al. (2011).

The data on the flooded depth and distance from the coastal line for the 18 sites in Table 3 may be represented and correlated as shown in Fig. 10. Although the number of sites in Table 3 is not large enough, the data on these sites indicate that the flooded depth decreases with an increase in the distance

Table 3
Flooded depth and its reduction.

No.	City	Observed site	Distance from coastal line X (m)	Flooded depth H (m)
1	Sendai	Arahama (Sea Port and Air Port Inst.)	223	4.4
2		Arahama Junior School (Shibayama etc.)	720	5.05
3		Bohken Plaza, High Bank	350	10.55
4		Higashiura, House	650	3.5
5		Takenohana at Sendai Tobu Highway	2400	1.6
6	Natori	Yuriage, Hiyoriyama	600	8.65
7		Yuriage Fishing Port (Shibayama etc.)	440	8.3
8		Sendai Air Port (Shibayama etc.)	1120	2.98
9	Iwanuma	Iwamuma Coastal Green Zone, Baseball Ground	350	6.7
10		Iwamuma Coastal Green Zone, View Stage	300	6.8
11		Fujisone, House	900	3.7
12		Iwamuma Coastal Green Zone, Office House	800	3.9
13	Watari	Torinoumi Hotel	270	5.0
14		Fisshing House	600	4.6
15		West Side of Fishing Port, House	810	3.9
16		Box 29 at Sendai Tobu Highway	2450	1.5
17	Yamamoto	Box 10 at Sendai Tobu Highway	3150	0.8
18		Nakahama Junior School	400	9.1

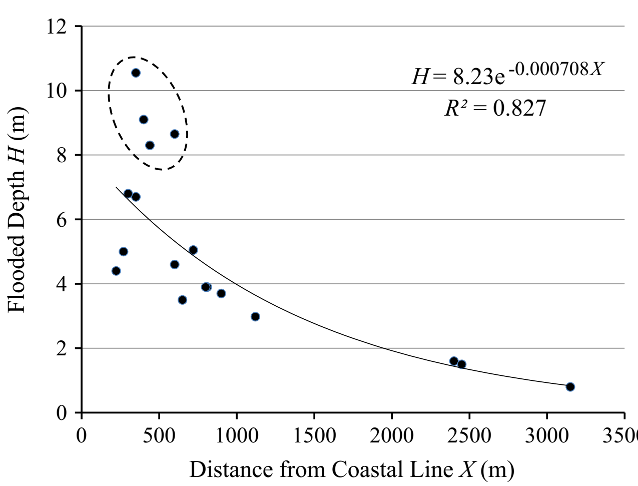

Fig. 10. Relation between X and H in case of average.

from the coastal line. This correlation may be represented by Eq. (6).

$$H = 8.23 \times \mathrm{EXP}(-0.000728 \times X) \quad (6)$$

where H=flooded depth (m), X=distance from the coastal line (m), and R^2=0.827.

In Fig. 10, it is seen that there are sites where the flooded depth is greater than that estimated by Eq. (6). In other words, the flooded depth at the sites close to the coast is underestimated. It is worth mentioning that the flooded depth close to the coast is very important for planning and designing countermeasures against a tsunami. A flooded depth of 8.23 m, as estimated by Eq. (6), at the coastal line appears to be a little small, because the tsunami height on the Sendai Plane is generally considered to be about 10 m.

Assuming that the tsunami height is equal to the flooded depth, Eq. (6) may be generalized in the form shown in Eq. (7).

$$H = a \times \mathrm{EXP}(b \times X) \quad (7)$$

where a and b are constants, and a is the flooded height at the coastal line (X=0). The value of a may be within the range 10–15. The appropriate value for a is discussed in the following.

Table 4 shows the values for b and for the coefficients of correlation (R^2) of Eq. (7) for different values of a in the range 10–5 m. It is seen for a=10–12 m, which is similar to the conditions in Fig. 10, that the flooded depth of 8–9 m is not consistent with the correlation line. However, for a=14–15 m, as shown in Fig. 11, the flooded depth at the sites far from the coast line is underestimated.

Therefore, a value of 13 for a is considered in this study and Eq. (8) is established; the correlation line representing Eq. (8) is shown in Fig. 12.

$$H = 13 \times \mathrm{EXP}(-0.001 \times X) \quad (8)$$

According to Eq. (8), the estimated flooded depths at X=1000 and 2000 m are 4.8 and 1.76 m, respectively (see Table 4).

Considering a flooded depth at the coastal line of 13.0 m, the bi-linear relationship in Fig. 12 may be represented by Eqs. (9.1) and (9.2) as follows:

$$0 \leq X \leq 1000 \text{ m}$$
$$H = 13\text{–}0.009X \quad (9.1)$$

$$1000 \leq X \leq 3500 \text{ m}$$
$$H = 5.5\text{–}0.0015X \quad (9.2)$$

According to Eqs. (9.1) and (9.2), the estimated flooded depths at X of 1000 and 2000 m are 4 and 2.5 m, respectively (see Table 4).

Table 4
Relations between a, b and R^2 and typical flooded height H according to X.

Modified method	Constants		R^2	Distance from Coastal Line X (m)				
	a	b		250	500	1000	2000	3000
Fundamental	8.23	−0.000728	0.827	6.9	5.7	4.0	1.92	0.93
$a=10$	10	−0.000844	0.788	8.1	6.6	4.3	1.85	0.79
$a=11$	11	−0.000901	0.741	8.8	7.0	4.5	1.81	0.74
$a=12$	12	−0.000954	0.681	9.5	7.4	4.6	1.78	0.69
$a=13$	13	−0.001000	0.613	10.1	7.9	4.8	1.76	0.65
$a=14$	14	−0.001050	0.537	10.8	8.3	4.9	1.71	0.60
$a=15$	15	−0.001090	0.457	11.4	8.7	5.0	1.70	0.57
Eqs. (9.1) and (9.2)				10.8	8.5	4.0	2.50	1.00

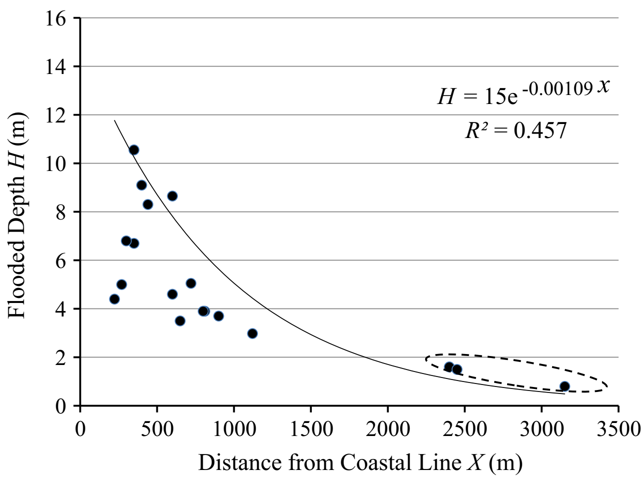

Fig. 11. Relation between X and H in case of $a=15$.

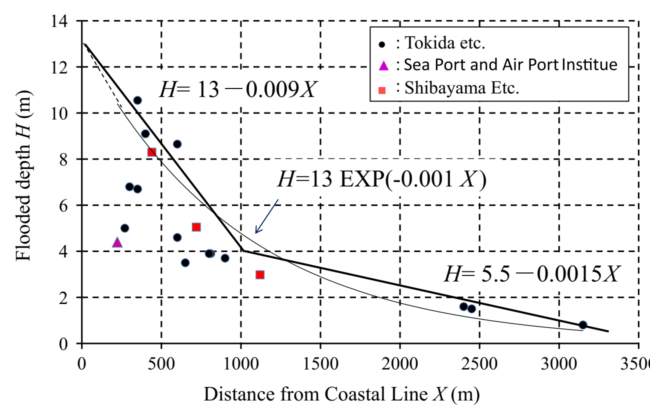

Fig. 12. Proposed relation between X and H.

Since Eqs. (9.1) and (9.2) is estimated on the basis of the data measured on the Sendai Plane, where the plane coast in general is almost straight and consists of a sand beach, sea wall, and forest, it can be used to roughly estimate the flood depth on the Sendai Plane for the case of the tsunami of March 11, 2011.

5. View on future hardware measures using earth structures

5.1. Applications based on learned lessons

On the basis of the above discussion, it is learned that structures such as earth banks, forests, and dug pools are effective in reducing the flood depth and/or the velocity of a tsunami, and therefore, can be used as effective hardware countermeasures against tsunami. The performance-based design approach for these structures in case of tsunami disasters is discussed in this section.

5.2. Classification of tsunami

The tsunami height, flooded height, and run-up height are important tsunami criteria, especially the flooded depth, because it is directly related to the damage of houses. In this study, the flooded depth is used, as shown in Table 5, to characterize the level of a tsunami into 3 categories: large, intermediate, and small. Flooded depths of 1.5 and 4 m are used to characterize whether the wooden houses are swept away or not. This is based on the research results of Iizuka and Matsutomi (2000). In this study, as shown in Table 5, tsunami heights of 4 and 8 m are estimated as almost twice the flooded depth.

The level of the tsunami may be identified as follows:

Large tsunami: More than two floors of houses are flooded and the houses are washed away. The height of the tsunami is greater than 8 m and the flooded depth is greater than 4 m.
Middle tsunami: The first and/or second floor of houses is flooded and the houses may be swept away. The height of the tsunami ranges from 4 to 8 m, and the flooded depth ranges from 1.5 to 4 m.
Small tsunami: The first floor may be flooded and the houses are not washed away. The tsunami height is lower than 4 m and the flooded depth is lower than 1.5 m.

5.3. Concept for performance-based method

To consider software and/or hardware countermeasures against future tsunami, principal approaches for resisting the tsunami should be clearly established. In this study, the approach for design is not only to save human lives, but also to protect assets such as houses. This design approach is performance-based and adopts actual objectives.

The approach of performance-based design against the tsunami is similar to that for other structures, such as for highway bridges that consider safety, serviceability, and recovery. However, when a tsunami countermeasure is designed, an additional design criterion, the protection of existing structures such as houses, should be considered.

The fundamental viewpoints on the performances necessary for the hardware countermeasures against tsunami may be considered as follows:

View point 1: When the tsunami does not overflow, the hard countermeasure should be stable.
View point 2: When the tsunami overflows, the hard countermeasure should be stable, though it is damaged, but the damage level should be small.
In addition, the following performance should be considered to estimate the effects of the countermeasures against the tsunami.
View point 3: The damage of houses, because of an overflowing flood, should satisfy the required level of the objective performance.

For the performance-based design, the performance levels that quantitatively consider the flood level and the damage level of wooden houses may be proposed as follows:

Class 1: Not flooded.
Class 2: Flooded under the level of the first floor. For example, the upper limit of flooded depth is approximately 0.5 m.
Class 3: Flooded below 1 m above the floor and the house is not destroyed to the middle level or swept away. The upper limit of the flooded depth is 1.5 m and the upper limit of the water flow velocity is 4.2 m/s.
Class 4: Flooded over the first floor and the house is destroyed to the middle and/or large level and/or washed away. The flooded depth is greater than 1.5 m and the water flow velocity is greater than 4.2 m/s.

This classification is summarized in Table 6. The classification of the middle and large levels of houses, the flooded depth of 1.5 m, and the water flow velocity of 4.2 m/s are based on the research results reported by Iizuka and Matsutomi (2000).

A comparison between Tables 5 and 6 implies that Classes 1–4 correspond to small level and middle or large level tsunami, respectively.

5.4. *Concept for hardware countermeasures*

The design concepts for the hardware countermeasure against tsunami that is performance-based may be proposed as follows:

Concept 1: The principal concept is that the tsunami should be contained in water areas such as the sea, a river, or a sea port. In other words, the flood by a tsunami into the inland is not permitted.
Concept 2: The containment of the tsunami is attained by means of a structure that can be a detached breakwater, sea wall, sand dune, or river levee. In addition to these structures, banks that have a reducing effect against tsunami (as discussed above) also contribute.
Concept 3: If it is difficult to contain the tsunami or the possibility of the tsunami is overestimated, the overflow of the tsunami is permitted.
Concept 4: The reducing performance of many kinds of hard countermeasures, such as banks, levees, forests, canals, and water zones, which are located between the sea coast and houses should be estimated.
Concept 5: The flood by the tsunami should be estimated based on the performance level of the wooden houses or the lightweight steel-frame houses considering the estimated indexes of flooded depth and water flow velocity which are highly related to the damage level of the houses.

Table 5
Classification of tsunami scale.

Scale of tsumami	Height of tsunami	Flooded depth	Remarks
Large	8 m≦	4 m≦	Coast at Senda Plane
Middle	4–8 m	1.5–4 m	Asahi City: Ashikawahama, Yasashigaura, Yokone, Shimonagai
Small	≦4 m	≦1.5 m	

Table 6
Performance class and identification based on flooded depth and flow velocity.

Performance class	Performance level	Flooded depth H (m)	Flow velocity u (m/s)
1	Not flooded	0	0
2	Flooded under the floor of first floor	$0 < H \leqq 0.5$	$0 < u \leqq 4.2$
3	Flooded under the 1 m high above the floor and house is not destroyed in the top level and flown away.	$0.5 < H \leqq 1.5$	
4	Flooded over the first floor and house is destroyed in the top and/or large level and/or flown away.	$1.5 < H$	$4.2 < u$

Note: Identification of the top and large level of house, flooded depth of 1.5 m and water flow velocity of 4.2 m/s are referred from Iizuka and Matsutomi (2000).

Concept 6: The permitted performance for the houses should be designed by selecting from Classes 1 to 3 which are shown in Table 6; these are the classes for the houses which are not destroyed severely, or washed away.

The above proposal is tentative at present; it must be checked and improved, so that damage due to the March 11 earthquake can be recovered and damage induced by future tsunami can be reduced.

It may also be said that Concepts 1 and 2 correspond to Level 1 proposed by the Central Disaster Management Council (2011) and represent the fundamental concept for preventing frequently possible tsunami, whereas Concepts 3–5 correspond to Level 2 that is also proposed to envisage highly possible tsunami. However, Concept 6 is related to both Level 1 and Level 2.

5.5. Case study on earth structure against tsunami

The typical structures which are related to reducing the water depth and/or flow velocity of the tsunami and clarified in the field survey conducted on the Sendai Plane are detached breakwaters, natural reefs, sand beaches, lagoons, sea walls, forests, canals, river levees, manmade banks, and road embankments.

When the reducing function of each structure can be quantitatively estimated, the performance-based design concept shown in Fig. 13 may be generally proposed. As shown in Fig. 13, the input conditions of the tsunami at the coastal line, such as the run-up height and the flow velocity, are gradually reduced through each reducing structure to output conditions such as the required flooded depth and flow velocity. The structures considered in the above design can be selected and 3 examples are shown in Fig. 13.

The corresponding structures in Examples 1 to 3 (showed Examples 1–3 in Fig. 13) are various structures, sea wall/bank, and sea wall/bank/canal, respectively. In other words, the combination of various reducing structures against the tsunami may be considered to represent a form of multiple defenses.

In order to apply the design concept shown in Fig. 13, further work is necessary to investigate and to quantitatively estimate the reducing function of the selected structures.

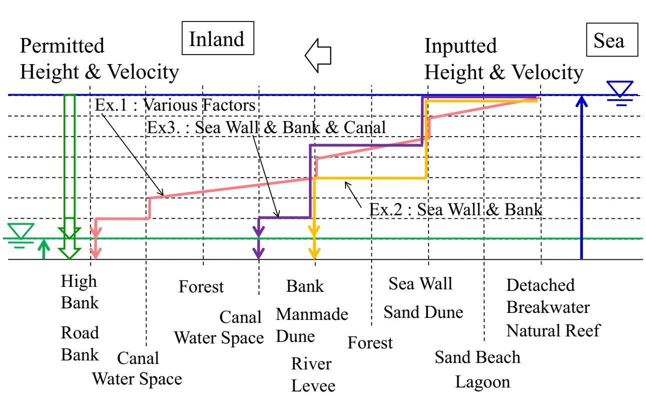

Fig. 13. Performance-based design concept combining factors related to reduction of tsunami.

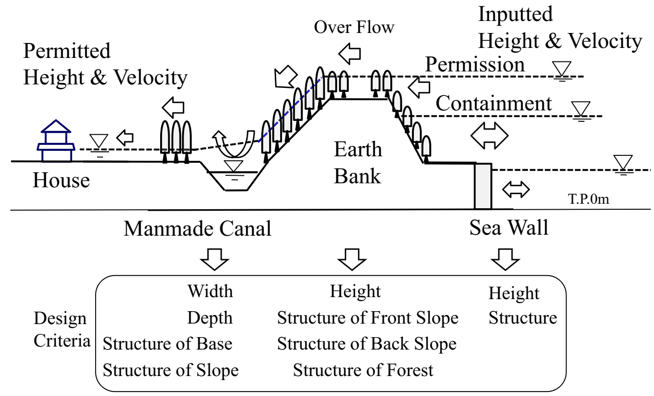

Fig. 14. Design concept against tsunami with sea wall, earth bank, and manmade canal.

In addition, considering the effects of the force reduction by earth banks, dug pools, and forests, as learned from the lessons in this paper, a combination of sea walls, earth banks with plants, and manmade canals can be proposed in case the tsunami overflow is permitted at the sea wall, as shown in Fig. 14 and/or Example 3 in Fig. 13; a manmade canal is constructed in advance instead of a dug pool. From the viewpoint of the performance-based design, each performance of the sea wall, the earth bank, and the manmade canal will be applied to reduce the depth and/or velocity of the tsunami flood to perform the Classes 1–3 in Table 6, and then the objective wooden houses located inland will be protected and not be swept away completely by future tsunami of Level 2.

6. Conclusions

In this paper, the resistance characteristics of earth structures, such as banks and dug pools, have been investigated and discussed based on the field surveys and other references such as the satellite photographs. The following lessons on the hard countermeasures with the use of the earth structures for the future recovery and reconstruction against tsunami damage can be indicated.

1) The 13 earth structures: 9 examples on the Sendai Plain in Miyagi Prefecture, 3 examples along the Asahi Coast in Chiba Prefecture, and 1 example along the Oarai Coast in Ibaraki Prefecture, such as the banks and so on which show us the effectiveness against the tsunami attacks, can be found based on the field surveys by the authors and others.
2) Earth structures which are covered with many kinds of materials, such as soils, asphalt, lawn, grass, and low or high trees, are eroded only at the surface and are difficult to completely destroy when the height of the earth structure is larger than about 4 m. On the other hand, earth structures lower than about 1.5 m can be washed away when the overflowed depth of the tsunami is larger than 3–4 m.
3) When the tsunami flows over the earth structures, it is easy to make dug pools at the back side of them. However, the tsunami attack can be reduced by dug pools, and furthermore, forests and/or houses at the back side of the dug pools can be protected from the tsunami. Structures similar

to dug pools, such as canals and water areas, located at the back side of earth structures, seem to be effective for protection against the tsunami flood.

4) 10 dug pools are selected and investigated and the relations between the width, the depth, the section area, and the height of the back slope are investigated. Equations to estimate their relations simply and roughly on the Sendai Plane in this- time tsunami can be proposed.
5) The ground surface at the background of the banks or sea walls seems to be eroded when the ground surface is not solid and/or covered with plants, and the sediment after the tsunami can be observed when the dug pool is a little large, for example, in 4.6 m of the eroded depth.
6) The flooded depth of the tsunami is reduced according to the distance from the coastal line, and the their relation can be proposed with the simple equations which can be applied roughly on the Sendai Plane in this-time tsunami.
7) For considering hardware countermeasures against tsunami in the future, performance-based design concepts are very necessary and effective for reducing tsunami damage according to the required damage level.
8) Earth structures, such as earth banks, should provide seismic stability against seismic motion before the tsunami attacks, and perform the tsunami reduction against the tsunami attacks combined with other structures, in other words, considering multiple defenses to satisfy the required performance of the objectives protecting against the overflow by the tsunami.

Future research subjects should include the detailed investigation of the resistant characteristics of earth structures through experiments and/or numerical analyses, and proposals of effective and economical hardware countermeasures against future tsunami using the resistance of the earth structures.

Acknowledgments

The authors would like to express their sincere gratitude to Mr. Wenzhong Chen of Nippo Co. Ltd., Assistant Professor Keigo Koizumi, and Mr. Tsuyoshi Akita of the Graduate School of Engineering of Osaka University who joined the field survey on the Sendai Plain, and Associate Professor Adel Mohamed El-Kelesh of Zagazig University in Egypt for technical support.

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